Unusual Necking of Cast-*in-situ* Concrete Piles

Rétrécissement Peu Commun de Pieux de Béton Moulés dans le Sol

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Summary

The investigations of a recent failure of a cast-*in-situ* pile foundation by necking near the top of the piles has drawn attention to the importance of a thorough site investigation even in built-up areas. A theory has been advanced attributing the necking to the combined effect of an upward flow of artesian water which delayed the set of the concrete, and a layer of fine sand below the critical density into which the liquid concrete penetrated.

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Les recherches concernant un accident récent survenu à un pieu moulé dans le sol et qui se rétrécit près du sommet du fût, ont attiré l'attention sur l'importance d'un examen minutieux de l'emplacement, même dans des agglomérations urbaines.

L'on a mis en avant une théorie qui attribue ce rétrécissement à l'effet combiné d'un mouvement ascensionnel d'eau artésienne qui aurait retardé la prise du béton, et d'une couche de sable fin, au dessous de la densité critique, dans lequel le béton liquide aurait pénétré.

A cast-*in-situ* concrete pile foundation in Durban, carried out by a reputable piling firm in accordance with the recommended practice regarding the sequence and separation distances of the newly formed piles, failed by necking of the pile shafts. This experience has disclosed two additional critical conditions affecting this type of construction: (a) an upward flow of water along the pile shafts has the effect of retarding the setting of the freshly placed concrete; and (b) where opportunities of volume change in the penetrated strata exist the possibility of the displacement of the delayed setting concrete from the pile shaft into the surrounding compressible strata should be considered.

Both factors emphasize the need for thorough site exploration in these cases even when a sound point bearing stratum is well established. The main features to be watched for are natural or induced artesian pressures and loose sand conditions below the water table.

The site, 150 × 113 ft., was originally the head of a creek which was partially filled in about 50 years ago. Prior to piling operations the site was levelled leaving a bank about 10 ft. above the excavation level at the west or landward side of the site while the spoil was dumped on the east or seaward side. The average soil profile of the site was 9 ft. of uniform sand fill, 27 ft. of medium fine to fine sands of varying density and colour and containing varying percentages of the finer fractions, overlying a dolerite dyke in the early stages of decomposition as indicated in Fig. 1. Ground water conditions were artesian, the piezometric head varying from 6 ft. above excavation level at the west side of the site to 4 ft. at the cofferdam later sunk near the east side. In a 14 in. test bore put down on the west side of the site some years earlier the ground water rose to the top of the casing which was estimated to be about 2 ft. 6 in. above the present excavation level, but as no additional casing was fitted the exact piezometric head was not determined.

The foundation consisted of 130 displacement piles and 20 bored piles. The displacement piles were formed by driving a 14 in. diameter closed end tube to the rock, placing the reinforcement and concrete, filling with sufficient excess to form an 18 in. diameter pile 36 ft. long and then withdrawing the tube with an up-and-down movement. No residual head of concrete was maintained in any of the pile tubes and no topping-up appeared to be necessary. On completion of the work the piles were to all appearances satisfactory, there being no indication that anything was amiss other than a slight boiling in the liquid cement laitance at the top of some of the piles immediately after the tube was withdrawn. The concrete consisted of a nominal 1:2:4 mix with $\frac{7}{8}$ in. coarse aggregate and of such consistency to ensure a solid pile. Of the 150 completed piles 80 were examined by the engineer, and all of these except for two groups of bored piles were found to be necked in varying

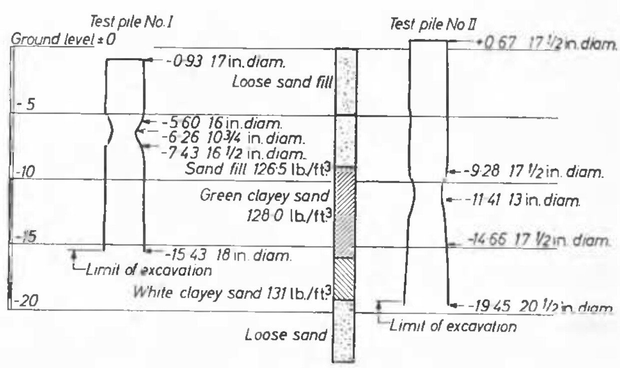

Fig. 1 Typical profiles
Coupes perpendiculaires types

degrees of severity. This necking started just below the top of the pile as shown in Figs. 1 and 2, the diameter being reduced to 7 to 10 in.—in many instances the reinforcing steel being exposed. The loss of concrete varied from $\frac{1}{4}$ to 3 cu. ft., the appearance of the neck being smooth in some cases and rough with exposed aggregate in others, particularly where the loss of concrete was large. The residue of laitance observed was 10 to 15 in. which is about 3 times as much as is normally found on the top of this type of pile.

Since difficult ground water conditions made it impossible to dig down to locate the seat of the trouble a test pile was made on the east side of the site; this was then surrounded by a 9 ft. square steel sheet pile cofferdam 31 ft. deep, i.e. to within about 5 ft. of the dolerite. A second test pile was then made inside the cofferdam, and boiling occurred immediately on removal of the tube but stopped after about 2 minutes. The concrete subsided about 15 in. leaving a large head of laitance.

Four Dutch probe tests were made, one at the cofferdam, each being accompanied by nearby test bores. All these probe tests which covered the whole site revealed a layer of very loose sand 5 ft. thick which varied between 18 and 26 ft., below the excavation level. In this layer the point resistance fell to zero ($\phi_a = 0$) whereas immediately above the layer the average value of ϕ_a was about 25 degrees, and below the layer the strength increased rapidly as the rock was approached. Excavation from the cofferdam proceeded by hand down to the surface of the loose sand layer when a blow-in occurred which stopped further operations. Undisturbed samples were taken at various depths, the results being as follows:

Depth	Soil	*Sat: unit* Wt. lb./cu. ft.	D_{10} in.	*U*	*k* cm/sec.
0–5	Loose sand fill	—	0·007	1·7	0·04 (by Hazen)
5–9	Denser sand fill	126·5	0·007	1·7	0·0005
9–16	Green clayey sand	128	0·005	1·8	$9{\cdot}75 \times 10^{-6}$
16–19	Fine white clayey sand	131	0·005	1·5	$< 9{\cdot}75 \times 10^{-6}$
19–	Fine loose sand	118 (Estimated)	0·005	1·5	0·003*

* Disturbed sample below critical density

The green clayey sand proved to have an unconfined shear strength of 2·35 lb./sq. in., part of which was attributed to capillary forces. The sand fill when saturated exhibited no apparent cohesion, and even the natural soil tended to collapse from the sides of a 4-in. auger hole after a short time, but this may have been due to the inflow of water along veins of clean grey sand which ran haphazardly through the green clayey sand. Due to the blow-in the loose sand layer could not be studied *in situ*, but the probe tests had shown that this sand was above the critical porosity which by shear tests on dry sand was found to be 45·5 per cent. The porosity of this sand by deposition through water was about 56 per cent and shaking the container reduced the porosity to only 51 per cent. The lowest value of the porosity obtained by vibration under a vertical load was 38 per cent. The driving of the sheet piling reduced the level of the soil inside the cofferdam by about 5 in. This was attributed to a settlement or change of volume within the loose sand layer, which, if the original porosity was taken as 52 per cent, would reduce the porosity of the 5 ft. layer to 48·5 per cent, that is still above the value of the critical porosity.

Both test piles were found to be necked, Figs. 1 and 2, but whereas No. 1 was typical in that the necking occurred near the top of the pile in the pervious fill, in No. 2 the necking was much lower down in the green sand and was followed by a thickening of the shaft which continued to the bottom of the excavation. A ring of fine light brown sand was observed to surround this pile in the region of the neck, and it appeared that as the tube was withdrawn this sand was forced up the void between the green clayey sand and the concrete by the upward flow of water. It is not unlikely that altered soil conditions were responsible for this change of behaviour since no washed-in material was found around the first test pile.

To account for the necking near the top of the piles in the zone of the more permeable soil rather than a general slump of concrete which would have been immediately apparent the following theory was advanced:

(a) On withdrawal of the tube, water under pressure from the loose sand layer escaped up the pile shaft. The bulk of this flow was probably around the perimeter of the shaft, and in some cases actually reached the top where boiling occurred for a few minutes after completion of the operation. The main flow of water was probably directly into the permeable sand fill some distance below the top of the pile. A small general slump of concrete may have occurred at this stage as the top of the aggregate was in all cases covered by a large thickness of laitance.

(b) This upward flow of water increased the water content of the concrete and by agitating the cement particles delayed the set. In the test piles the concrete dried very white indeed, indicating a high water-cement ratio.

(c) The concrete in the upper part of the pile shaft no longer or never, as the case may be, subjected to the flow of water gained its initial set and became strong enough to grip the reinforcement and the sides of the hole, any downward movement thus being prevented. In the lower sections of the pile shaft

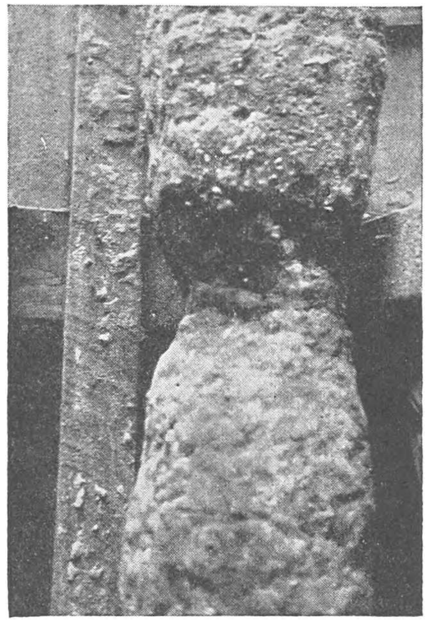

Fig. 2 Necking of concrete pile
Etranglement du pieu en béton

the setting of the concrete was very much delayed by the effect of the upward flowing water.

(d) The walls of the hole lubricated by the flow of water allowed the liquid unset concrete to settle bodily downward as displacement into the layer of loose sand occurred, at the same time drawing a neck just below the partially set concrete in the upper part of the shaft. This appeared to be a slow process, as in the case of test pile No. 1, where the concrete in the neck had a comparatively smooth surface. Alternatively, a faster run, of perimeter concrete only, occurred over a much longer length leaving the concrete retained by the steel reinforcement cage with a very rough outward appearance. The weak sand fill completed the process by collapsing into the void so formed.

The validity of this theory depends upon two factors: (1) the set of the concrete due to an upward flow of water being sufficiently delayed to enable the above mechanism to occur; and (2) a satisfactory explanation of the entry of the liquid concrete into the loose sand layer.

In order to investigate the first factor setting tests were made on samples of concrete grout under an upward flow of water, needle penetrations taken at various intervals being compared with those made on control samples of the same mix and water contents which were not subjected to any flow of water. In a test on a 1:2 mix with a water-cement ratio of 52 per cent by weight the control sample set in about 3 hours (*Note:* South African cements have a much longer setting time than English cements), by which time the sample under flow had taken only about three-sevenths of its set and was still in a plastic state. In a second test with a water-cement ratio of 45 per cent the control pat set in 80 minutes by which time the sample under flow had taken only one-sixth of its set.

With the wetter mixes used on the site and with the increase of water content due to the entry of water there could be no doubt that the setting time of the concrete lower down in the shafts was greatly delayed beyond that of the concrete at the top of the pile shafts.

Regarding the second factor two separate cases had to be considered, the smooth drawn neck and the longer and rougher perimeter run. The quake due to driving adjacent pile tubes was undoubtedly an important factor, but to a larger extent fortuitous depending on the distance between the new pile and the last formed pile and on the time elapsed. These factors probably accounted for some piles having smooth necks (case of the influence of the pile itself) and others having rougher necks (case of the additional influence of driving adjacent pile tubes).

Since no further piles were driven until No. 1 test pile was completely set, this pile was considered as typical of the first case, that is, piles finished at the end of the days' work or beyond the influence of quake. Two possible causes of the entry of the concrete into the loose sand layer were the disturbance due to the gentle vibratory extraction of the tube and the radial compression of the loose sand layer by the lateral pressure of the liquid concrete once the tube was withdrawn. With the concrete transformed into a liquid state by the upward flow of artesian water the effective lateral pressure of a column of concrete 22 ft. 6 in. long was taken to be 1650 lb./sq. ft. This pressure would cause radial compression in the loose sand layer which was estimated on the basis of the effective pressure-void ratio relationships determined in the laboratory assuming the pressure to vary inversely as the distance from the pile. For sand having an initial porosity of 49·4 per cent the compression was estimated as 0·16 in. which over a 5 ft. layer was equivalent to 0·25 cu. ft. This was approximately the amount of concrete lost in the neck of No. 1 test pile.

It was considered not unlikely that the rough perimeter runs were due to the quake effect superimposed upon the causes just discussed, but unfortunately no evaluation was possible in these cases. It is, however, well known that a violent shock or tremor will cause loose sand to contract in volume with momentary increase of neutral stress, the amount of contraction depending upon the type and severity of the shock and upon the void ratio and fineness of the sand. It would appear that all of these necessary conditions were satisfied during the installation of the foundation and that as a consequence unusually large volume changes occurred. These conditions were probably aggravated in the vicinity of one group of bored piles where exceptionally heavy loss of ground occurred during the sinking of the 14 in. casings.

An attempt was made to remove the two test piles in order to see what had happened in the loose sand layer, but unfortunately only the top halves of the piles were recovered. The usual remedy in cases where the pile shaft has to intersect layers of loose sand below the water table lies in forming the pile inside a light metal casing which remains part of the finished pile —whether this should be full length or need only overlap the dangerous zone depends upon such local conditions as the thickness of the zone, the properties of the adjacent soil, the water pressure and its source. A metal casing with its top 2 or 3 ft. above the top of the loose sand layer would undoubtedly have prevented the necking in the case under discussion. The percolation of water up the perimeter of the shaft above the casing would have continued, however, but with no effect other than a possible weakening of the concrete leading to low early strength. If this is undesirable the casing should be carried up the full length of the pile shaft.

This experience illustrated the necessity for a thorough investigation of subsoil and ground water conditions even in built-up areas, particularly in estuarine localities dominated by a rising hinterland, such as exist along the Natal coast. The importance of the Dutch probe as an essential piece of investigation apparatus was also clearly demonstrated, and there can be very little doubt that had this apparatus been used in the initial investigation the danger might have been foreseen and guarded against.

In conclusion it would be well to point out that with this combination of circumstances, a very loose sand layer and an upward flow of artesian water, necking may well occur even if special efforts are made to eliminate disturbance of the loose sand layer. The two test piles graphically illustrated this point. On this site it was fortunate indeed that the necking occurred above pile cut off; had the permeable sand fill been a few feet deeper the necking would have probably been lower down the shafts and would never have been discovered.